STORMWATER TREATMENT AREA NO. 3 & 4

PLAN FORMULATION DOCUMENT

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8. CANAL AND STRUCTURE HYDRAULICS

8.1 INTRODUCTION

This section of the *Plan Formulation Document* provides information on the hydraulic design criteria for the primary canals and structures associated with STA-3/4. The primary canals and structures include G-371 on the North New River Canal and G-373 on the Miami Canal; supply canal, inflow canal and inflow control structures; and discharge canal, L-5 Canal and outflow control structures.

For G-371 and G-373, hydraulic analyses were conducted of both the North New River and Miami canals to establish design headwater and tailwater elevations to satisfy conveyance requirements at the two structures. Additionally, this section includes definition of the hydraulic criteria for stilling basin design at both structures.

For the supply canal and other inflow works, a one-dimensional HEC-RAS model was utilized to evaluate the capability of the facilities to convey water at rates and with profiles consistent with the effective use of the proposed pump station and canal capacity. The hydraulic evaluation balanced the required canal conveyance capacity of the supply canal section with the desire to minimize excavation requirements.

HEC-RAS was also used to model the proposed distribution of STA outflows through the discharge canal and to the L-5 Borrow Canal and downstream works, consistent with that distribution recommended in the *Alternatives Analysis*. These model simulations have been conducted in conjunction with the 2D modeling efforts in order to establish discharge boundary conditions.

8.1.1 Manning's n Value

The one-dimensional HEC-RAS models for the canal hydraulics were developed and simulated with both design and conservative loss parameters. The design value, as implied, serves as the basis of design for all canals associated with STA-3/4. The conservative value is utilized for evaluation of the results to determine the potential range (i.e., worst case) of operation of the system. The design manning's n value is assigned as 0.023 and the conservative manning's n value is assigned as 0.028. These values are applicable for straight and uniform earth channels, free of aquatic vegetation. Additionally, these manning's n values are consistent with those determined in previous studies of canal hydraulics in south Florida including:

- U.S. Army Corps of Engineers, Central and Southern Florida Project, Part VI General Studies and Reports, Section 5 Design Memorandum Channel Roughness, 1953.
- Burns & McDonnell, Supplemental Analysis of the L-3 Borrow Canal, Stormwater Treatment Area No. 5, 1999.

8.2 STRUCTURES G-371 AND G-373

Structure G-371 will be located in the North New River Canal approximately 1500 ft upstream (north) of existing Pump Station S-7 and will be used to bypass regulatory flows from Lake Okeechobee and flood flows around STA 3/4. Since S-7 serves as full-time outflow pump station for STA 3/4, G-371 will not be used to bypass flood flows unless the pump station is off line or a disastrous flood occurs.

Structure G-373 will be located in the Miami Canal approximately 7 miles downstream of new Pump Station G-372 and serves to facilitate bypassing of regulatory releases from Lake Okeechobee and flood flows around STA 3/4. G-373 will not be used to bypass flood flows unless the pump station S-8 is off line or a disastrous flood occurs.

The purpose of the hydraulic analyses for these two structures is to establish the design headwater and tailwater elevations to satisfy the conveyance requirements at the two structures. From this information hydraulic design criteria for Structures G-371 and G-373 can be developed.

The U.S. Army Corps of Engineers (COE) HEC-2 model was utilized for hydraulic analyses of structures G-371 and G-373. The design concept is to determine the allowable stage increases at G-371 and G-373 which would not result in the COE design water surface elevations at the upstream end of the canals at Lake Okeechobee being exceeded. The hydraulic analysis was then performed for regulatory release flow rates of 1600 cfs and 2000 cfs (the design capacities of the North New River and Miami canals under regulatory release) for Structure G-371 and G-373, respectively.

8.2.1 Canal Hydraulic Analyses

The HEC-2 files used in this study are based on the files used in *Lower East Coast Regional Storage Transfer Alternatives* study performed for SFWMD (Gee & Jenson, 1994). These files were updated with twelve cross sections for North New River and Miami Canal obtained in 1999, as discussed in Section 4 of the September 1999 *Alternatives Analysis*. The HEC-2 files for the 1994 study used cross-section data obtained from as-built cross-sections of the original excavation dated circa 1957 and the Hump Removal excavation dated circa 1978.

The design of the structures is based on the requirement of not increasing stages at Lake Okeechobee above the COE design stages for regulatory releases. According to the General Design Memorandum for the L-18, L-19 (North New River Canal), L-23 and L-24 (Miami Canal) Hump Removal, the COE design elevations at the Lake Okeechobee are 13.2’ for the Miami Canal and 13.3’ for the North New River Canal. In this study the maximum upstream water surface elevations used in hydraulic analysis for the structure design are 13.2 ft for Miami Canal and 12.85 ft for North New River Canal. The lower elevation of 12.85 ft for the North New River Canal was used in order to limit potentially objectionable stage increases.

For regulatory flows, a tailwater elevation of 9.5 ft at S-7 and S-8 was provided for this study, consistent with the original design basis for the C & SF Project. The HEC-2 model simulations for the North New River Canal and Miami Canal were performed from S-7 and S-8 to the upstream ends of the canals at the Lake Okeechobee respectively. For flood flows, a tailwater elevation of 10 ft at S-7 and S-8 was provided for this study. Flood flow rates were assigned at 2,170 cfs and 3,670 cfs in the North New River and Miami canals, respectively, equal to the established design capacities of new inflow pumps in stations G-370 and G-372.

A summary of the hydraulic analysis performed for structure G-371 and G-373 is presented below:

North New River Canal

Regulatory release flow rate (1600 cfs)

The HEC-2 model was initially run without G-371 with a tailwater of 9.5 ft at S-7. The water surface elevation at the upstream end of the canal at Lake Okeechobee was calculated to be 12.55 ft.

The HEC-2 model was then run to determine the allowable head loss for G-371. A head loss of 0.5 ft for G-371 resulted in a stage of 12.85 ft at Lake Okeechobee.

Therefore, a head loss of 0.5 ft will be used as the design criteria for structure G-371.

Flood Flow Rate (2170 cfs)

The HEC-2 model was run with a preliminary size for G-371 in order to find the head loss of the structure under the flood flow conditions. The tailwater and headwater at G-371 were calculated to be 10.04 ft and 10.88 ft, respectively. The simulation results showed that the head loss for structure G-371 was approximately 0.84 ft.

Miami Canal

Regulatory release flow rate (2000 cfs)

The HEC-2 model was initially run without G-373 and a tailwater of 9.5 ft at S-8. The water surface elevation at the upstream end of the canal at Lake Okeechobee was calculated to be 13.10 ft.

The HEC-2 model was then run to determine the allowable head loss for G-373. A head loss of 0.2 ft for G-373 resulted in a stage of 13.20 ft at Lake Okeechobee. Therefore, a head loss of 0.2 ft will be used as the design criteria for structure G-373.

Flood Flow Rate (3670 cfs)

The HEC-2 model was run with a preliminary size for G-373 in order to find the head loss of the structure under the flood flow conditions. The tailwater and headwater at G-373 were calculated to be 11.40 ft and 11.85 ft, respectively. The simulation results showed that the head loss for structure G-373 was approximately 0.45 ft.

The hydraulic grade lines from the hydraulic analysis for regulatory flows are plotted in Figures 8.1 and 8.2.

8.2.2 Limiting Conditions for Stilling Basin (Energy Dissipation) Design

Preliminary gate configurations for G-371 and G-373 were developed based on the allowable head losses for regulatory flows. The overall design of energy dissipation (stilling basins) and structure protection for G-371 and G-373 will be based upon a sudden gate opening under maximum probable head differential. The maximum probable head differential at G-371 is taken as 3.1 feet. That differential is equal to the maximum historic headwater elevation at S-7 of 13.1 ft. NGVD and the normally maintained canal elevation of 10.0 ft. NGVD downstream of the structure. The maximum probable head differential at G-373 is taken as 3.7 feet. That differential is equal to the difference between the maximum historic headwater elevation at S-8 of 13.7 ft. NGVD, and the normally maintained canal elevation of 10.0 ft. NGVD downstream of the structure.

It is anticipated that the sudden full opening of the structure gates under those head differentials may result in flows exceeding the safe (e.g., non-scouring) capacities of the receiving canals. While it is considered prudent to design the stilling basins and structure protection for those potential rates of flow, the detailed design of the two structures

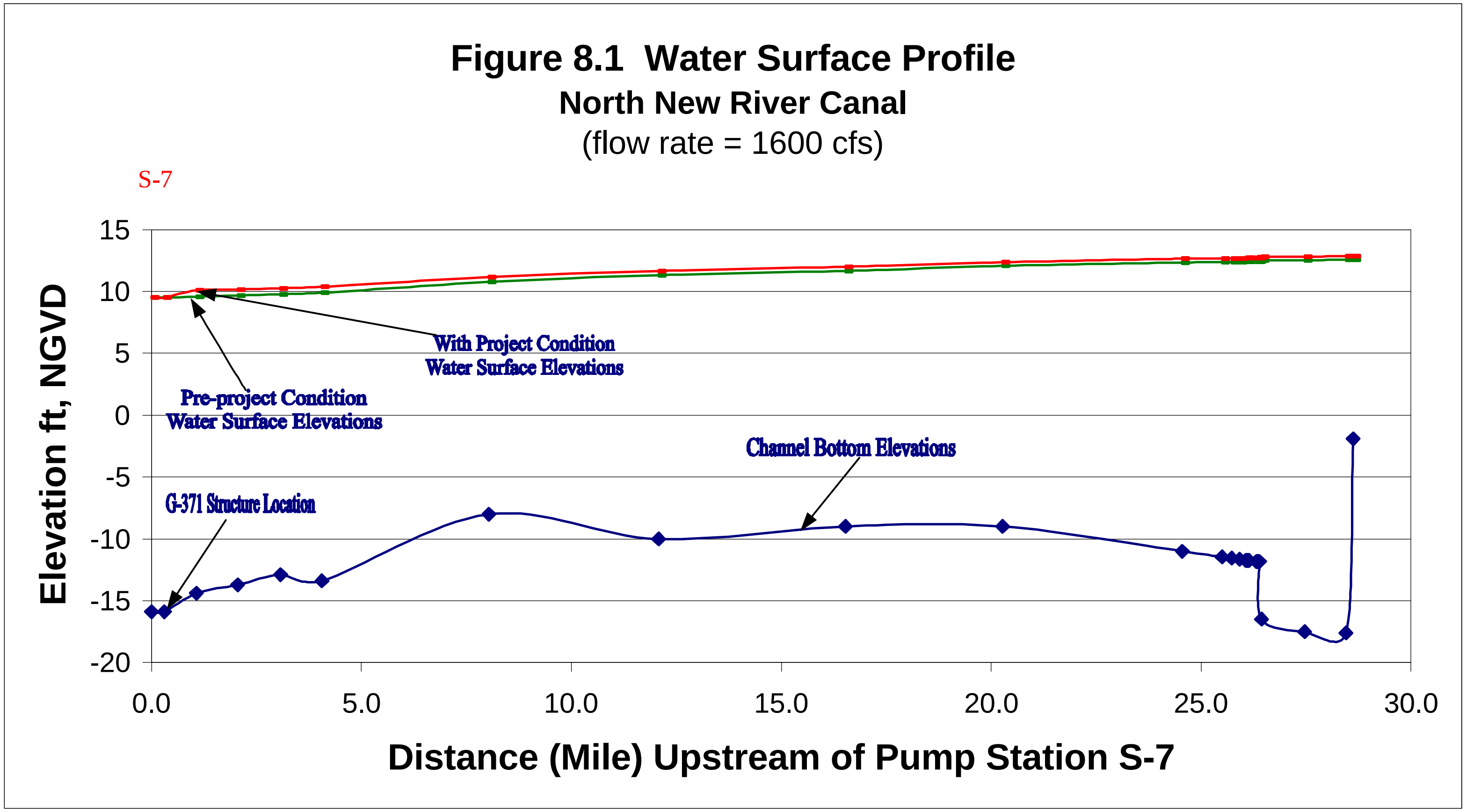

Figure 8.1 Water Surface Profile
North New River Canal
(flow rate = 1600 cfs)
S-7
15
10
5
0
-5
-10
-15
-20
Elevation ft, NGVD
With Project Condition
Water Surface Elevations
Pre-project Condition
Water Surface Elevations
Channel Bottom Elevations
G-371 Structure Location
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Distance (Mile) Upstream of Pump Station S-7

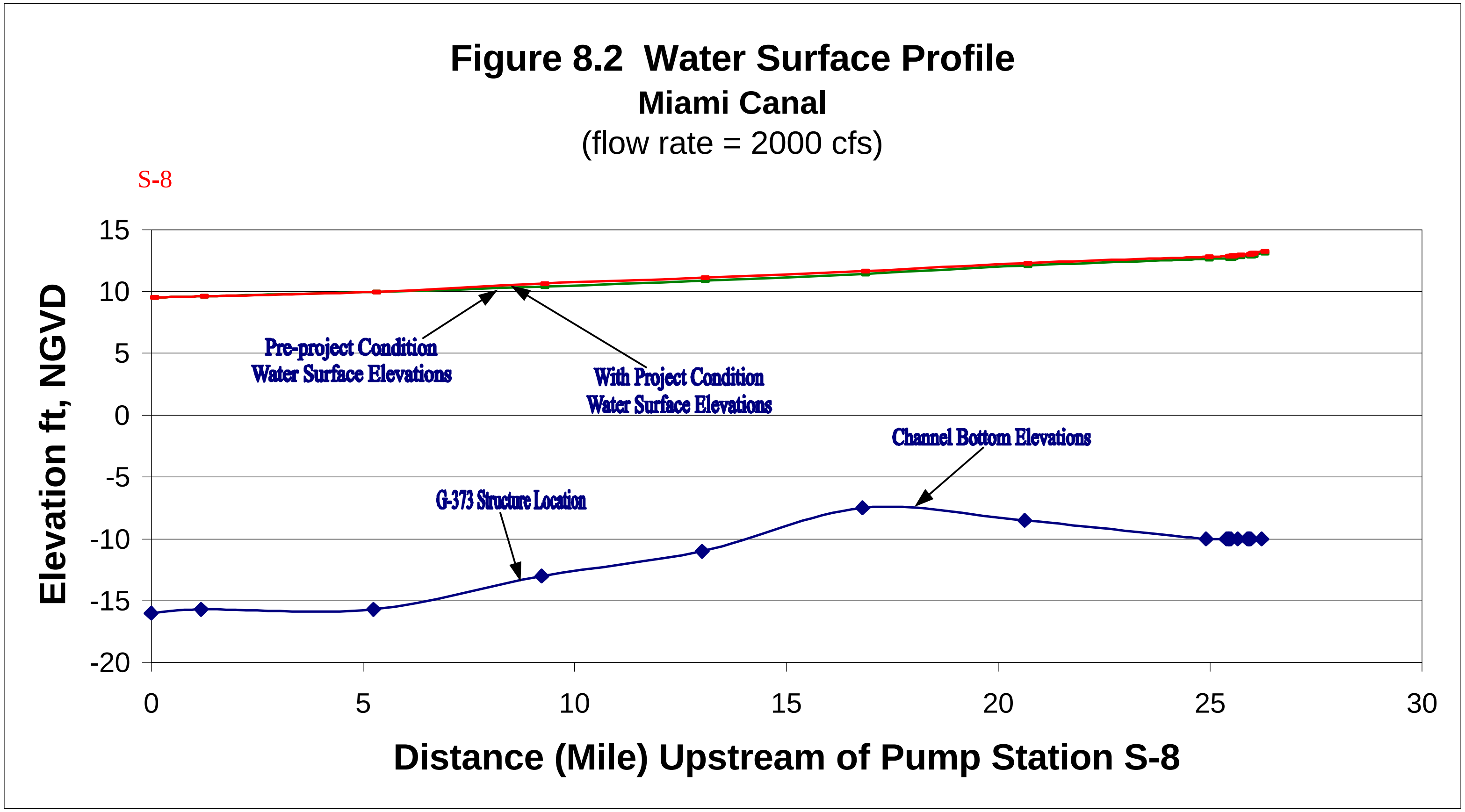
Figure 8.2 Water Surface Profile
Miami Canal
(flow rate = 2000 cfs)
S-8
Elevation ft, NGVD
Distance (Mile) Upstream of Pump Station S-8
Pre-project Condition
Water Surface Elevations
With Project Condition
Water Surface Elevations
Channel Bottom Elevations
G-373 Structure Location
15
10
5
0
-5
-10
-15
-20
0
5
10
15
20
25
30

should include specific recommendations for limiting gate openings as may be necessary to limit velocities in the downstream canals to 2.5 fps or less.

Based on recently conducted cross section surveys at the locations of these structures, the maximum allowable (controlled) release rate at G-371 is 3,380 cfs. This high rate of discharge would of necessity be limited in duration, as the nominal capacity of S-7 is 2,490 cfs. The maximum allowable (controlled) release rate at G-373 is 3,670 cfs (coincidentally equal to the design rate of flood flow of that same location).

8.2.3 Summary of Hydraulic Design Criteria

The recommended hydraulic design criteria for the structures are summarized in the Table 8.1.

Table 8.1

Hydraulic Design Criteria

Diversion Structures G-371 & G-373

Description	Units	G-371	G-373
Gate Capacity			
Discharge	cfs	1,600	2,000
Headwater Elevation	ft. NGVD	10.03	10.19
Tailwater Elevation	ft. NGVD	9.52	9.98
Energy Dissipation			
Discharge	Maximum	Capacity	Capacity
Headwater Elevation	ft. NGVD	13.2	13.7
Tailwater Elevation	ft. NGVD	10.0	10.0
Controlled Gate Openings			
Discharge	cfs	3,380	3,670
Headwater Elevation	cfs	Variable	Variable
Tailwater Elevation	ft. NGVD	10	10
Full Bypass			
Discharge	cfs	2,170	3,670
Headwater Elevation	ft. NGVD	10.8	11.9
Tailwater Elevation	ft. NGVD	10.0	11.4

8.3 SUPPLY CANAL AND INFLOW WORKS

Inflows to STA-3/4 will be delivered from both the Miami Canal and the North New River Canal. Water from the Miami Canal will be lifted by Pump Station G-372 and conveyed to the east by the supply canal along the north side of the Holey Land. At the northwest corner of STA-3/4, the supply canal transitions into the Cell 3 and Cell 2 inflow canal and continues along the north line of STA-3/4. Similarly, water from the North New River Canal will be lifted by Pump Station G-370 and conveyed to the west via the Cell 1 inflow canal.

For the supply canal and inflow canal, a one-dimensional HEC-RAS analysis was conducted to evaluate water surface profiles and resulting channel velocities at even pump increments for pumping station G-372 and G-370. For G-372, the pumping station is anticipated to consist of 4 - 925 cfs pumps resulting in flow rates of 3670 cfs, 2775 cfs, 1850 cfs, and 925 cfs corresponding to the operation of 4, 3, 2 and 1 pump(s), respectively. For G-370, the pumping station is anticipated to consist of 3 - 725 cfs pumps resulting in flow rates of 2175 cfs, 1450 cfs, and 725 cfs corresponding to the operation of 3, 2 and 1 pump(s), respectively.

Two separate HEC-RAS models were developed as follows:

- For the Supply Canal and Cells 2 and 3 Inflow Canal, extending from Structure G-383 between Cells 1 and 2 (i.e., the downstream end) to Pumping Station G-372 located just east of the Miami Canal.
- For the Cell 1 Inflow Canal, extending from Structure G-383 between Cells 1 and 2 (i.e., the downstream end) to Pumping Station G-370 located just west of the North New River Canal.

The supply canal has a bottom width of 45 feet with 2.5H:1V side slopes with a depth of approximately 15 feet. The supply canal is flanked by the enlarged Holey Land levee to the south and the perimeter levee / maintenance berm to the north. The typical section for the supply canal, both from the Miami Canal to P.S. G-372 and from P.S. G-372 to the NW corner of STA-3/4, is included in Part 1 of this *Plan Formulation Document* as Plate 11.

The inflow canal ranges in bottom width from a minimum of 14 ft to a maximum of 45 ft depending on the peak flow rate in the canal. The canal has 2.5H:1V side slopes and a depth of approximately 15 feet. The inflow canal is flanked by the perimeter levee / access berm to the north and the inflow control levee to the south. The typical sections for the inflow canal are included in Part 1 of this *Plan Formulation Document* on Plates 9 and 10.

Detailed plan view drawings in the vicinity of G-370 and G-372 are also included in Part 1 of this *Plan Formulation Document* on Plates 3 and 4, respectively.

From this analysis, the estimated tailwater stages at G-372 and G-370 were computed as a function of pump station discharge, starting Water Surface Elevation (WSE) at G-383 (located in the inflow canal at the northwest corner of Cell 1A, see Plate 1, in Section 1), and manning's n value. The results are included as Figures 8.3 and 8.4 and Tables 8.2 and 8.3. Additionally, a summary of the headwater stages for each of the inflow control structures is included in Section 8.7 and Table 8.11 of this report. These stages are based on the design (i.e., maximum) inflow to STA-3/4.

Figure 8.3

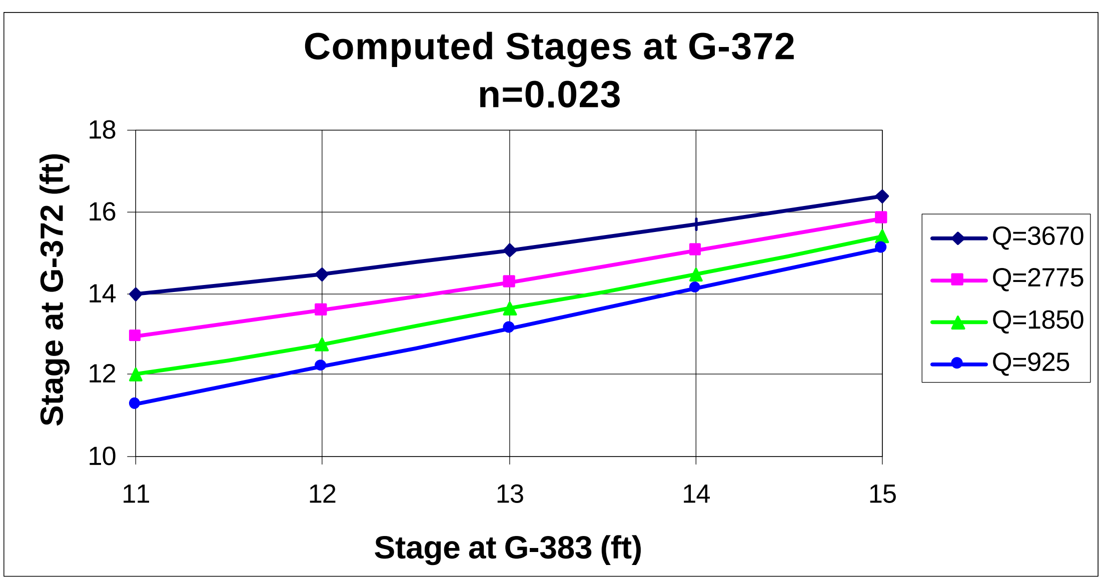

Table 8.2

Computed Stages at G-372

Starting WSE at G-383	Tailwater Stage at G-372 Manning's n = 0.023 / n = 0.028			
	Discharge Rate (cfs)			
	3670	**2775**	**1850**	**925**
11	13.96/14.86	12.94/13.61	11.99/12.37	11.27/11.39
12	14.45/15.26	13.57/14.15	12.77/13.09	12.21/12.30
13	15.03/15.75	14.27/14.77	13.61/13.87	13.16/13.24
14	15.69/16.32	15.04/15.46	14.49/14.70	14.13/14.19
15	16.40/16.95	15.85/16.21	15.39/15.57	15.10/15.15

Figure 8.4

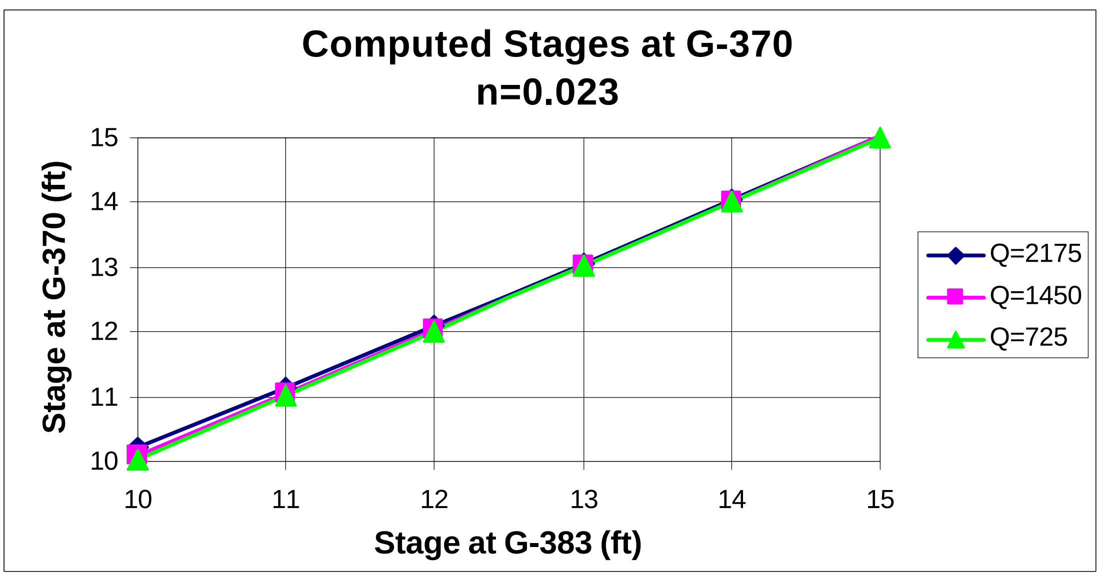

Table 8.3

Computed Stages at G-370

Starting WSE at G-383	**Tailwater Stage at G-370 Manning's n = 0.023 / n = 0.028**		
	Discharge Rate (cfs)		
	2175	**1450**	**725**
10	10.21/10.33	10.09/10.15	10.02/10.04
11	11.14/11.24	11.06/11.11	11.02/11.03
12	12.10/12.17	12.05/12.08	12.01/12.02
13	13.07/13.12	13.03/13.06	13.01/13.01
14	14.05/14.09	14.02/14.04	14.01/14.01
15	15.04/15.07	15.02/15.03	15.00/15.01

8.4 OUTFLOW WORKS

After passing from north to south through STA-3/4, the outflow will be collected in a discharge canal along the south side of the STA. From this discharge canal, the treated discharges will be conveyed along the L-5 Borrow Canal to the west to the Miami Canal (to P.S. S-8) or to the east to the North New River Canal (to P.S. S-7) and WCA-3A (via S-150). The existing L-5 Borrow Canal will be enlarged east and west of STA-3/4 discharge canal to provide adequate conveyance capacity for the maximum anticipated discharges from STA-3/4.

Several HEC-RAS models were developed to simulate the proposed distribution of STA outflows through the discharge canal and to the L-5 Borrow Canal and downstream works (i.e., to S-7 on the NNR Canal and to S-8 on the Miami Canal). Four separate HEC-RAS models were developed as follows:

- For the West L-5, extending from the S-8 (i.e., downstream end) to the western rock pit on the Griffin property. The energy gradient at the most upstream cross section is assumed to be the resulting water surface elevation in the rock pits.
- For the Discharge Canal, extending from the easternmost rock pit on the Griffin property (i.e., downstream end) to the southeast corner of STA-3/4. This model is used in conjunction with that for the West L-5 (i.e., the resulting WSE in the rock pits from the West L-5 model is utilized as the starting boundary condition for this model) to estimate stages in the discharge canal east of the rock pits.

- For the East L-5, starting at S-7 (i.e., downstream end) through the discharge canal to the eastern rock pit on the Griffin property.
- For the Cell 3 Discharge Canal, extending from the southwest corner of Cell 3 adjacent to the Holey Land to the western rock pit on the Griffin property.

These hydraulic analyses were conducted for inflow rates to STA-3/4 based on 20% increments of inflow pump station capacity for G-370 and G-372. Cell 1 of STA-3/4 is intended to convey and treat the inflow from G-370 on the North New River. Similarly, the inflow from G-372 on the Miami Canal is distributed between Cells 2 and 3 on the basis of effective treatment area, or roughly 54% and 46% of the total from G-372, respectively.

The primary hydraulic constraint for the distribution of outflows from STA-3/4 is the maximum allowable headwater and tailwater elevations at the outflow control structures. These elevations will vary by cell and are determined in part from the 2D modeling of the treatment cells. These target elevations are summarized in Tables 8.4, 8.5 and 8.6.

Table 8.4

Hydraulic Constraints for Cell 1B

% Pump Capacity	Total Flow in Cell 1	Estimated HW (1)	Estimated Headloss (2)	Estimated Maximum TW
100	2170	12.8	0.50	12.3
80	1736	12.5	0.32	12.2
60	1302	12.2	0.18	12.0
40	868	11.8	0.08	11.7
20	434	11.2	0.02	11.2
Mean Annual	398	11.2	0.01	11.2

(1) The estimated HW for 100% pump capacity was determined during the conduct of the 2D modeling based on the specific maximum allowable depth criteria for this cell. Other estimates of HW are calculated based on $Q=kd^{2.6}$ with k=87.38.

(2) The estimated headloss through the outflow control structures is based on the orifice equation $Q=CA(2gh)^{½}$ given 0.5 ft of headloss at 100% pump capacity.

Table 8.5

Hydraulic Constraints for Cell 2B

% Pump Capacity	Total Flow in Cell 2B [(1)]	Estimated HW [(2)]	Estimated Headloss [(3)]	Estimated Maximum TW
100	1980	12.9	0.30	12.6
80	1578	12.6	0.19	12.4
60	1183	12.3	0.11	12.2
40	789	11.9	0.05	11.9
20	394	11.4	0.01	11.4
Mean Annual	263	11.1	0.01	11.1

(1) This is the flow in Cell 2B assuming that the STA is operated with 3 parallel flow paths, not cells in series.
(2) The estimated HW for 100% pump capacity was determined during the conduct of the 2D modeling based on the specific maximum allowable depth criteria for this cell. Other estimates of HW are calculated based on $Q=kd^{2.6}$ with k=88.82.
(3) The estimated headloss through the outflow control structures is based on the orifice equation $Q=CA(2gh)^{½}$ given 0.3 ft of headloss at 100% pump capacity.

Table 8.6

Hydraulic Constraints for Cell 3

% Pump Capacity	Total Flow in Cell 3	Estimated HW [(1)]	Estimated Headloss [(2)]	Estimated Maximum TW
100	1690	13.5	0.50	13.0
80	1358	13.3	0.32	13.0
60	1019	13.1	0.18	12.9
40	679	12.8	0.08	12.8
20	340	12.4	0.02	12.4
Mean Annual	224	11.0	0.01	12.2

(1) The estimated HW for 100% pump capacity was determined during the conduct of the 2D modeling based on the specific maximum allowable depth criteria for this cell. Other estimates of HW are calculated based on $Q=kd^{1.78}$ with k=140.4.
(2) The estimated headloss through the outflow control structure, G-382, is based on the orifice equation $Q=CA(2gh)^{½}$ given 0.5 ft of headloss at 100% pump capacity.

8.4.1 West L-5 Borrow Canal

The recommended enlargement to the west L-5 Borrow Canal is shown in Part 1 of this *Plan Formulation Document* on Plate 8. In general, the canal invert is lowered to elevation –5.0 ft NGVD, or approximately 5.0 ft, and the bottom width is widened by 24 ft with 2.5H:1V side slopes. Plate 8 also shows the preliminary typical section for the discharge canal for STA-3/4. The discharge canal has a bottom width of 18 ft, invert elevation at -7 ft NGVD and 2.5H:1V side slopes. Plan view drawings of the west L-5 improvements in the vicinity of existing pumping stations G-201 and S-8 are included on Plates 6 and 7, respectively.

The HEC-RAS models for the West L-5 and the Discharge Canal were utilized to determine the maximum feasible flow to the west given this enlarged configuration of the L-5 Canal, the typical section of the discharge canal and the limiting water surface elevations at the outflow control structures. This analysis was conducted at 20% increments of total inflow pump station capacity. The results are summarized in Figure 8.5 and Tables 8.7 and 8.8.

Figure 8.5

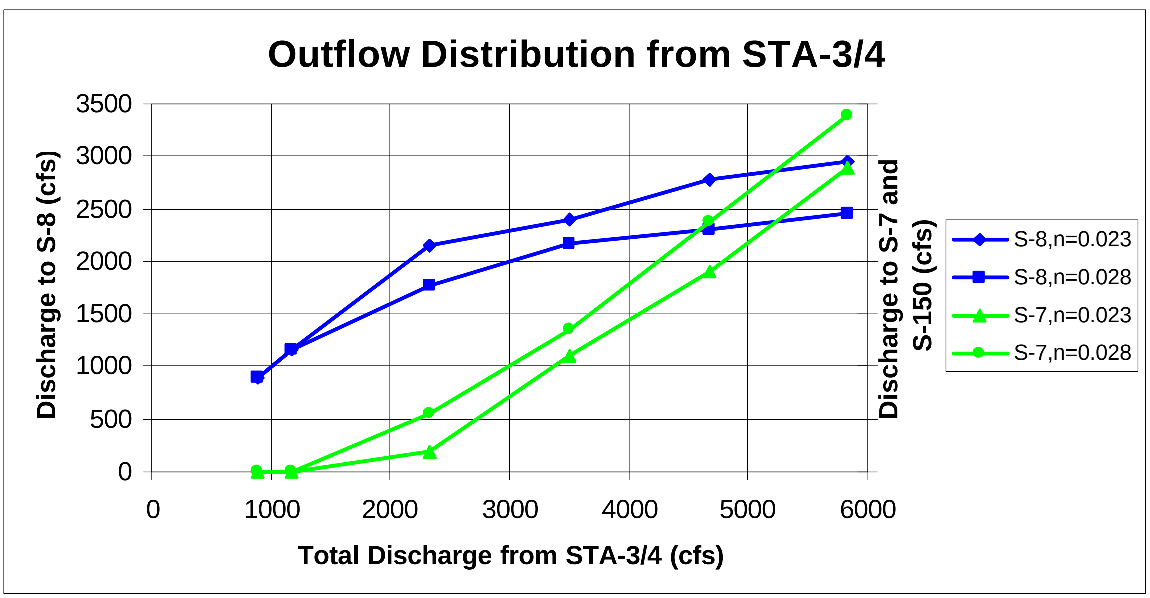

Table 8.7

Maximum Discharge to the West

Design Loss Parameters (n=0.023)

% Pump Capacity	**Total Q (cfs) into STA-3/4**	**Max Q (cfs) to West in L-5**	**Peak WSE (ft, NGVD)**	**Location of Peak WSE**
100	5840	2950	12.60	Cell 2 / G-379B
80	4672	2770	12.39	Cell 2 / G-379A
60	3504	2400	11.94	Cell 1 / G-376F
40	2336	2150	11.69	Cell 1 / G-376B
20	1168	1168	10.58	Cell 1 / G-376A
Mean Annual	885	885	10.36	Cell 1 / G-376A

Table 8.8

Maximum Discharge to the West

Conservative Loss Parameters (n=0.028)

% Pump Capacity	**Total Q (cfs) into STA-3/4**	**Max Q (cfs) West L-5**	**Change in Q (cfs)**	**Peak WSE (ft, NGVD)**	**Location of Peak WSE**
100	5840	2450	500	12.60	Rock Pit
80	4672	2300	470	12.36	Cell 2/G-379C
60	3504	2160	240	12.18	Cell 2/G-379A
40	2336	1775	375	11.63	Cell 1/G-376D
20	1168	1168	0	10.83	Cell 1/G-376A
Mean Annual	885	885	0	10.51	Cell 1/G-376A

From this analysis, it is determined that the target outflow distribution from STA-3/4 at peak flows can be conveyed given this preliminary design for modifications to the L-5

Canal. Additionally, at approximately 20% of the total inflow pumping capacity or 1170 cfs, all flows can be directed to the west to Pumping Station S-8.

8.4.2 East L-5 Borrow Canal

The recommended enlargement to the east L-5 Borrow Canal is shown in Part 1 of this *Plan Formulation Document* on Plate 8. In general, the canal invert is maintained at elevation –3.5 ft NGVD and the bottom width is enlarged to a total width of 60 ft. The canal enlargement is anticipated to have 2.0H:1V side slopes, except in the vicinity of the existing Highway 27 bridges where the side slope is transitioned to 1.5H:1V to match the existing cross section beneath the bridges. A plan view drawing of the east L-5 improvements in the vicinity of existing pumping station G- S-7 is included on Plate 5.

Given the maximum feasible discharge to the west, the East L-5 canal was evaluated to determine the maximum allowable headwater elevation at S-7. This analysis was based on meeting the estimated WSE in the discharge canal (from the results of the West L-5 analysis) at the location of the flow split from west to east. For this analysis, S-7 was utilized for any discharge up to 2490 cfs; all discharges greater than 2490 cfs were directed to S-150. . The results are summarized in Figure 8.6 and Tables 8.9 and 8.10.

Figure 8.6

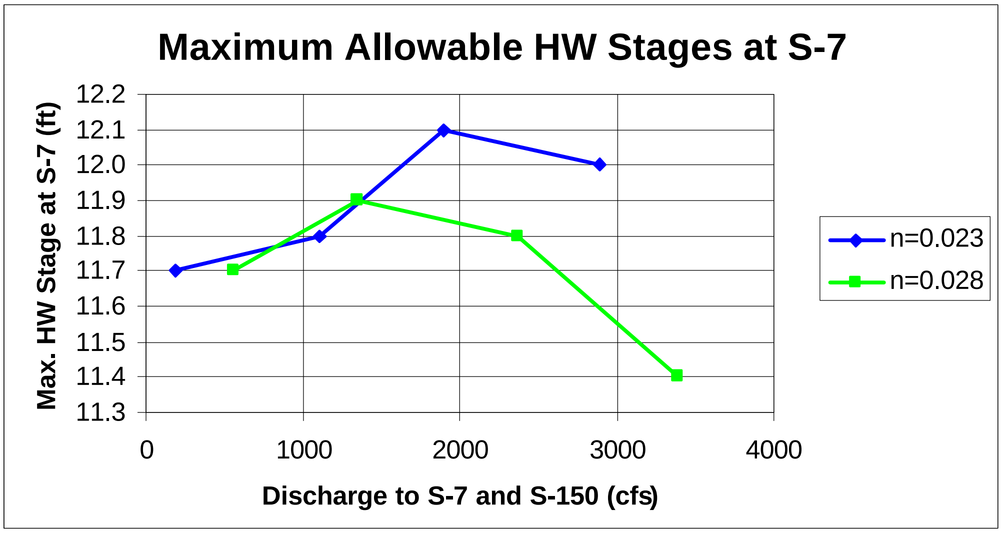

Table 8.9

Allowable HW at S-7

Design Loss Parameters (n=0.023)

% Pump Capacity	Total Q (cfs) into STA-3/4	Est. Q (cfs) East L-5	Max HW (ft, NGVD) at S-7	Peak WSE (ft, NGVD)	Location of Peak WSE
100	5840	2890*	12.0	12.5	Cell 2/G-379B
80	4672	1902	12.1	12.3	Cell 2/G-379A
60	3504	1104	11.8	11.9	Cell 1/G-376F
40	2336	186	11.7	11.7	Cell 1/G-376B
20	1168	0	NA	NA	NA
Mean Annual	885	0	NA	NA	NA

*2490 cfs to S-7, remaining 400 cfs to S-150

Table 8.10

Allowable HW at S-7

Conservative Loss Parameters (n=0.028)

% Pump Capacity	Total Q (cfs) into STA-3/4	Est. Q (cfs) East L-5	Max HW (ft, NGVD) at S-7	Peak WSE (ft, NGVD)	Location of Peak WSE
100	5840	3390*	11.4	12.6	Rock Pit
80	4672	2372	11.8	12.3	Cell 2/G-379C
60	3504	1344	11.9	12.1	Cell 1/G-379A
40	2336	561	11.7	11.7	Cell 1/G-376D
20	1168	0	NA	NA	NA
Mean Annual	885	0	NA	NA	NA

*2490 cfs to S-7, remaining 900 cfs to S-150

8.4.3 Comparison of Projected Stage-Duration to Historic Levels

Historic headwater stage durations at Pumping Stations S-7 and S-8 and Structure S-150 were taken from the District's data files. The S-7 and S-8 data were retrieved for the period including calendar years 1965 through 2000 and 1962 through 2000, respectively. Information at S-7 was taken from DBKEY 04330 (for years 1965-85) and DBKEY 06695 (for years 1985-2000). Information at S-8 was taken from DBKEY 04342 (for years 1962-85) and DBKEY 06697 (for years 1985-2000). The S-150 data were retrieved for the period including calendar years 1994 through 2000. Information at S-150 was taken from DBKEY 16558.

It should be noted that historic headwater durations at S-150 fall somewhat below those at S-7. This is believed to result primarily from the differing periods of record considered, as headwater elevations at these two structures should be relatively consistent given their close physical proximity and direct hydraulic connection of headwaters.

Figures 8.8 through 8.10 present the projected stage durations in the L-5 Borrow Canal for S-7, S-8, and S-150. These stage-durations are applicable to the period 1965-95, and were developed specifically for the intentional delivery of maximum discharge volumes from STA-3/4 to pumping station S-8. They were also developed for operating scenario 3 as it is presented in Section 7, and would increase slightly for operating scenario 2.

For S-7 Headwater, projected stage durations were obtained using discharge durations from Figure 7.11 (Scenario 3) coupled with the Maximum Headwater elevations shown in Table 8.9. For total STA-3/4 discharges below 3000 cfs, the stage at S-7 will be essentially the same as the Rock Pit (Figure 8.7). Figure 8.8 shows these historic and projected stage durations at S-7 Headwater. The projected stage durations of S-150 are the same as S-7 data because of its close proximity, and are shown with the historic headwater durations for S-150 in Figure 8.9.

At S-8, the projected operating level ranges from 10.0 to 10.7 ft. NGVD except when stage levels falls below 10.0 ft. NGVD as shown in Figure 8.10. Drawdowns below elevation 10.7 ft, NGVD are assumed to follow the level of Cell 1, as shown in Figure 7.8.

When considering the entire period of record as at S-7 and S-8, it is clear that stage durations in L-5 will fall below historic values. Comparison to more recent operating levels at S-150 suggests a relatively close parallel between historic and projected stage durations. Additional comparisons along West L-5 at G-204, G-205 and G-206 are included in Section 10 of this *Plan Formulation document*.

It is concluded on the basis of this analysis that no adverse impact to facilities (such as FPL transmission lines) adjacent to the L-5 Borrow Canal will result from stage changes in L-5 due to construction and operation of STA-3/4.

Insert Figure 8.7 here

Insert Figure 8.8 here

Insert Figure 8.9 here

Insert Figure 8.10 here

8.4.4 Cell 3 Discharge Canal

It is recommended that the existing Holey Land seepage canal be enlarged to serve as the Cell 3 Discharge Canal. The recommended enlargement is shown in Part 1 of this *Plan Formulation Document* on Plate 12. In general, the canal enlargement has a lowered invert elevation at –6.5 ft NGVD, bottom width of 10 feet, and 2.5H:1V side slopes in locations with a discharge of 1690 cfs (i.e., the design flow rate). This applies to that reach of the canal that extends from the easternmost outflow control structure to the outfall into the rock pits. For flows less than 1690 cfs, the canal enlargement has a lowered invert elevation at –1.0 ft NGVD, bottom width of 10 feet, and 2.5H:1V side slopes. This applies to the discharge canal generally along the south boundary of Cell 3.

The HEC-RAS model was utilized to determine the tailwater elevations at the outflow control structures for Cell 3. The starting water surface elevation was based on that estimated in the rock pits (from the HEC-RAS model for the West L-5 Canal) given the maximum feasible flow to the west. This analysis was conducted at 20% increments of total inflow pump station capacity. The results under peak rates of flow are summarized in Table 8.11.

8.5 NORTH PERIMETER SEEPAGE CANAL ANALYSIS

A UNET model was developed for evaluation of the seepage canal along the northern perimeter of STA-3/4 from Pumping Station G-372 to Pumping Station G-370. The seepage canal is a trapezoidal canal with a bottom width of 10 ft, an invert elevation of 0.0 ft NGVD and 2H:1V side slopes; this section is consistent section along the entire 16-mile length of the canal. A manning's n value of 0.045 is utilized representing a relatively straight and uniform channel, constructed in earth with heavy aquatic growth. This value is taken from Appendix B of the Design of Small Dams, Bureau of Land Reclamation.

The seepage canal was separated into approximately 2-mile segments for defining the lateral inflow from seepage from the supply canal through the perimeter levee. The rate of seepage inflow is as determined from the analysis of seepage at the site (presented in

Part 4 of this *Plan Formulation Document*, Table 4.13). The values utilized were 33.0 cubic feet per day/foot of length/foot of head along the Supply Canal and 35.4 cubic feet per day/foot of length/foot of head along the inflow canal. For computation of the head differential, the water surface elevation in the supply canal and inflow canal is taken as the average elevation for each 2-mile segment as determined from the HEC-RAS model for the design inflow condition and a starting WSE of 15 at G-383. The elevation in the seepage canal is assigned as 6.5 ft NGVD at the downstream end of the model at P.S. G-370. Additionally, a constant discharge of 151 cfs is defined at the upstream end of the model at P.S. G-372. Several iterations were conducted to compute the WSE in the seepage canal and then update the estimates of seepage based on the head differential between the Supply Canal and Inflow Canal.

The results from the UNET model indicate a maximum water surface elevation of 8.0 feet, located along the northeast corner of the Holey Land. This elevation is approximately 1.5 to 2.0 feet below the prevailing grade. Additionally, this analysis shows that the total seepage will be divided evenly between pumping stations at a rate of 151 cfs each. The design headwater elevation for the seepage pumps at G-370 is 6.5 feet, and the design headwater elevation for the seepage pumps at G-372 is 6.88 feet.

8.6 EAST PERIMETER SEEPAGE CANAL ANALYSIS

A perimeter levee and exterior seepage canal form the eastern boundary of STA-3/4 and are generally parallel to U.S. Highway 27. The seepage canal is a trapezoidal canal with a bottom width of 10 ft, an invert elevation of -3.0 ft NGVD and 2.5H:1V side slopes; this section is consistent along the entire 5.2-mile length of the canal. The typical section is shown in Part 1 of this *Plan Formulation Document* on Plate 12. A manning's n value of 0.045 is utilized representing a relatively straight and uniform channel, constructed in earth with heavy aquatic growth.

Seepage along the eastern perimeter was estimated for both the northern section adjacent to Cell 1A and the southern section adjacent to Cell 1B. The rate of seepage inflow is as determined from the analysis of seepage at the site (presented in Part 4 of this *Plan Formulation Document*, Table 4.13). The values utilized were 44.7 cubic feet per day/foot of length/foot of head for the East Perimeter North (i.e., Cell 1A) and 55.8 cubic feet per day/foot of length/foot of head for the East Perimeter South (i.e., Cell 1B). For computation of the head differential, the average water surface elevations for Cells 1A and 1B were estimated as 14.06 ft and 13.05 feet, respectively, from the 2D modeling for the design flow conditions. The water surface elevation in the adjacent seepage canal was based on the water surface elevation in the discharge canal at the point of inflow for the seepage canal for design flow conditions plus an allowance of 0.5 feet for headloss through the control structure. This results in an estimated water surface elevation of 12.6 feet. The total seepage along the East Perimeter North is estimated as 11 cfs, and the total seepage along the East Perimeter South is estimated as 4 cfs. The seepage along the northern perimeter of STA-3/4 conveyed to G-370 totals 151 cfs as determined from the UNET analysis.

A HEC-RAS model was developed to estimate the peak stages in this eastern seepage canal when all seepage collected at G-370 and along the eastern perimeter are directly discharged by gravity to the discharge canal at the southeastern corner of STA-3/4. It is estimated that a total of 166 cfs of seepage would be conveyed along the eastern perimeter of STA-3/4 under peak design conditions. The resulting peak water surface elevation at G-370 and generally adjacent to U.S. Highway 27 is 12.7 feet.

Actual stages will typically range between 10.0 and 11.0 ft. NGVD, with higher stages only under infrequent conditions. It is recommended that the potential for elimination of the east perimeter levee and seepage canal be considered during detailed design. The ability to delete those facilities will be controlled by:

- FDOT acceptance
- Whether or not direct discharge of recovered seepage is desired (see Section 9 of this *Plan Formulation document*).

8.7 SUMMARY OF CONTROL STRUCTURE HYDRAULIC ANALYSES

A summary of the various hydraulic analyses for the treatment area inflow and outlet control structures is presented in Table 8.11. This tabulation combines the results of the 2-D modeling discussed in Section 6 with the analyses presented herein.

Table 8.11

Summary of Control Structure Hydraulic Analyses

Structure			Design Discharge	Headwater	Tailwater
Inflow Control Structures	Cell 1:	G-374 A	362 cfs	15.01	14.32
		G-374 B		15.01	14.35
		G-374 C		15.00	14.38
		G-374 D		15.00	14.40
		G-374 E		15.00	14.43
		G-374 F		15.00	14.45
	Cell 2:	G-377 A	396 cfs	14.90	14.40
		G-377 B		14.90	14.41
		G-377 C		14.90	14.42
		G-377 D		14.90	14.42
		G-377 E		14.90	14.42
	Cell 3:	G-380 A	282 cfs	14.95	13.97
		G-380 B		15.01	13.98
		G-380 C		15.04	13.98
		G-380 D		15.10	13.99
		G-380 E		15.14	13.99
		G-380 F		15.18	13.99
Intermediate Control Structures	Cell 1:	G-375 A	362 cfs	13.81	13.20
		G-375 B		13.81	13.26
		G-375 C		13.81	13.29
		G-375 D		13.81	13.31
		G-375 E		13.81	13.33
		G-375 F		13.81	13.33
	Cell 2:	G-378 A	396 cfs	13.82	13.46
		G-378 B		13.82	13.49
		G-378 C		13.82	13.51
		G-378 D		13.82	13.52
		G-378 E		13.82	13.51
Outflow Control Structures	Cell 1:	G-376 A	362 cfs	12.81	12.12
		G-376 B		12.81	12.25
		G-376 C		12.81	12.35
		G-376 D		12.81	12.42
		G-376 E		12.81	12.47
		G-376 F		12.81	12.50
	Cell 2:	G-379 A	396 cfs	12.93	12.52
		G-379 B		12.92	12.53
		G-379 C		12.92	12.59
		G-379 D		12.91	12.59
		G-379 E		12.91	12.59
	Cell 3:	G-381 A	282 cfs	13.76	12.77
		G-381 B		13.76	12.89
		G-381 C		13.76	13.08
		G-381 D		13.76	13.17
		G-381 E		13.76	13.21
		G-381 F		13.76	13.23

8.8 SUMMARY OF HYDRAULIC DESIGN CRITERIA

This section summarizes the hydraulic design criteria of primary treatment area control structures, pumping stations and miscellaneous structures, other than G-371 and G-373 (see Table 8.1).

8.8.1 Primary Control Structures

This section summarizes the recommended hydraulic design criteria and normal operating levels at the various inflow and outflow control structures within STA-3/4. The hydraulic design criterion is based on the peak or design rate of discharge into STA-3/4, a total of 5840 cfs, and the analytical results discussed in Section 8.7. This inflow is proportioned to the three parallel flow paths on the basis of area, so that each flow path receives an equivalent hydraulic loading. The summary of the primary control structure hydraulic design criteria are presented in Table 8.12; this table also identifies the anticipated size of the structure barrel, expected to consist of a reinforced concrete box culvert.

Table 8.12

Hydraulic Design Criteria

Primary Control Structures, Design Discharge

Strucuture	Design Discharge	Headwater Ft., NGVD	Tailwater Ft., NGVD	Size W x H
Inflow Control Structures:				
Cell 1: G-374 A-F (6 structures)	362 cfs	14.9	14.4	10' x 8'
Cell 2: G-377 A-E (5 structures)	396 cfs	14.9	14.4	10' x 9'
Cell 3: G-380 A-F (6 structures)	282 cfs	15.1	14	7' x 7'
Intermediate Control Structures:				
Cell 1: G-375 A-F (6 structures)	362 cfs	13.8	13.3	10' x 8'
Cell 2: G-378 A-E (5 structures)	396 cfs	13.8	13.5	10' x 10'
Outflow Control Structures:				
Cell 1: G-376 A-F (6 structures)	362 cfs	12.8	12.3	10' x 8'
Cell 2: G-379 A-E (5 structures)	396 cfs	12.9	12.6	10' x 10'
Cell 3: G-381 A-F (6 structures)	282 cfs	13.8	13.1	8' x 8'

The normal operating levels is based on the average annual discharge into STA-3/4, a total of 885 cfs (approximately 641,000 acre-feet per year). This inflow is proportioned to the three parallel flow paths on the basis of area, so that each flow path receives an equivalent hydraulic loading. The summary of the primary control structure normal operating levels are presented in Table 8.13.

Table 8.13
Normal Operating Levels
Primary Control Structures

Strucuture	**Average Discharge**	**Headwater Ft., NGVD**	**Tailwater Ft., NGVD**
Inflow Control Structures:			
Cell 1: G-374 A-F (6 structures)	398	12.0	11.9
Cell 2: G-377 A-E (5 structures)	263	12.2	12.1
Cell 3: G-380 A-F (6 structures)	224	12.7	12.6
Intermediate Control Structures:			
Cell 1: G-375 A-F (6 structures)	398	11.5	11.5
Cell 2: G-378 A-E (5 structures)	263	11.9	11.9
Outflow Control Structures:			
Cell 1: G-376 A-F (6 structures)	398	11.2	11.1
Cell 2: G-379 A-E (5 structures)	263	11.1	11.0
Cell 3: G-381 A-F (6 structures)	224	12.5	12.4

8.8.2 Pumping Stations

The following are summaries of recommended design criteria for pump capacities, together with headwater (suction) and tailwater (discharge) stages resulting from the analyses previously presented in this Section 8.

Primary Pumping Units

The following (Table 8.14) are recommended hydraulic performance requirements for the primary (inflow) pumping units at pumping stations G-370 and G-372.

Table 8.14
Recommended Hydraulic Performance Requirements
Primary (Inflow) Pumping Units at G-370 and G372

ITEM	STATION G-370	STATION G-372
Pumped Capacity	2,170 cfs	3,670 cfs
Suction/Discharge Elevations		
Max. High Water Condition	14.0/18.0 NGVD	14.0/19.0 NGVD
Min. Low Water Condition	8.0/9.0 NGVD	8.0/9.0 NGVD
Design Flow Condition	10.0/13.0 NGVD	10.0/13.0 NGVD
Rated Flow Condition	10.0/15.0 NGVD	10.0/17.0 NGVD

Each station's maximum pump efficiency point at full flow will be established for the "design flow condition". Full flow capacity will also be required for the "rated flow condition". Each station will be expected to pump a minimum of 80% of full flow for the "low water" and "maximum high water" conditions.

- The "design flow condition" was selected to obtain maximum efficiency of the pumping units under headwater and tailwater conditions that can be expected the majority of the time, in the interest of overall fuel economy.
- The "maximum high water" conditions were selected as follows. On the headwater (suction) side of the stations, the maximum high water elevation of 14 was established slightly above the historic maximum headwater elevations at existing pumping stations S-7 and S-8 (13.2 and 13.8 ft. NGVD, respectively). The maximum high water elevation on the tailwater (discharge) side of the stations was established approximately three feet above the calculated water surface elevation associated with full rates of inflow against a stormwater treatment area at maximum design depth. That increase was selected as an allowance against the potential of an extreme rainfall

event occurring while the system is at capacity, and to assure that the mechanical design of the primary pumping units and associated drivers is sufficiently robust.

- The “minimum low water” conditions were selected to be somewhat below (approx. 1 ft.) the minimum levels actually anticipated. A degree of conservatism is warranted here to assure that adequate suction head is available to prevent cavitation, and to assure that the downstream ends of the pump discharge tubes remain submerged under all foreseeable pumping conditions. Should the tailwater elevation drop below the crown of the discharge tubes, the vacuum assist in the pumping station design would be lost, with resulting adverse impact on both pumping unit rate and the load applied to the mechanical systems.
- The “rated flow condition” was established to assure the availability of the required nominal capacity of the station when the STA is at hydraulic capacity.

Seepage Pumps

The maximum predicted rate of seepage return to each of Stations G-370 and G-372 is 150 cfs, as discussed previously in this Section 8. A total of three 75-cfs capacity vertical turbine pumps will be installed at each station to pump the seepage flow. Two pumps are designed to pump the 150-cfs flow with one pumping acting as a stand-by, or backup pump.

The final design water elevation conditions for the seepage canal and pumps are established as in Table 8.15.

Table 8.15
Final Design Water Elevations:
Seepage Canal and Pumps

ITEM	STATION G-370	STATION G-372
Flow Capacity	150 cfs	150 cfs
Suction/Discharge Conditions		
Design Water Elevations	6.9/13.0 NGVD	6.5/13.0 NGVD
Rated Water Elevations	6.9/15.0 NGVD	6.5/17.0 NGVD
Maximum Discharge Water Elevations	6.9/18 NGVD	6.5/19.0 NGVD

The pumps should deliver not less than 100% of design flow for both the “design” and “rated” conditions. The pumps should achieve the point of maximum efficiency when pumping under the “design water elevations” conditions.

The design headwater (suction) elevations for the pumps was established on the basis of the UNET analysis of the seepage collection canal discussed earlier in this report, and should be adequate to prevent stages at any point along the seepage canal under maximum design inflows from exceeding 8.0 ft. NGVD.

8.8.3 Miscellaneous Structures

The overall plan for construction of STA-3/4 includes five hydraulic control structures not intended for common use in controlling the distribution of treatment area inflows and outflows. The following paragraphs define the intended utilization of those structures and hydraulic criteria established for their design.

Structure G-383

This structure is situated in the Inflow Canal immediately north of the intersection of the Inflow Control Levee with Interior Levee 1. This normally closed structure serves to maintain separation between inflows from the North New River Canal and the Miami Canal, assuring the ability to direct all North New River inflows to Cell 1A, and all Miami Canal inflows to Cells 2A and 3. That basic function could be accomplished through construction of a simple earthen plug of the inflow canal at that same location. It is considered preferable to provide the capacity to direct interbasin exchanges of inflow in the interest of operational flexibility.

As a result, Structure G-383 is included primarily in the interest of operational flexibility; specific capacity requirements cannot be directly established. It has been assumed that this structure would operate at maximum capacity when all inflows to the STA are from one canal (either the North New River or the Miami), and it is desired to uniformly distribute those inflows to the entire treatment area. Approximately 60% of the total effective treatment area is in Cells 2 and 3. Under a peak inflow of 2,170 cfs from the North New River Canal, it might under some circumstance be desirable to pass 60% of that rate (1,300 cfs) through G-383 to Cells 2 and 3. Under a peak inflow of 3,670 cfs from the Miami Canal, it might also under some circumstance be desirable to pass 40% of that rate (1,470 cfs) through G-383 to Cell 1. The maximum design capacity of G-383 is therefore taken as 1,470 cfs. Additional discussion on the desirability of this structure is included in Section 9 of this *Plan Formulation document*.

Design headwater and tailwater elevations are presently indeterminate, but in any event can be manipulated by active operation of the various inflow control structures to achieve the desired flow distribution. A double 10’x10’ RCB would be capable of transferring a discharge of 1,470 cfs with a head differential of approximately 1.4 feet, and is recommended for this structure.

Structures G-382A and G-382B

Structure G-382A is situated in Interior Levee 1, and connects the collection canals at the downstream ends of Cells 1A and 2A. Structure G-382B is situated in Interior Levee 4, and connects the collection canals at the downstream ends of Cells 2A and 3. These structures are expected to normally remain closed, and are included in the design to increase operational flexibility. They would permit the inter-cell transfer of flows at an intermediate point in the overall flow path. One example of their potential use would be to improve overall treatment performance should it be determined during operation that a net benefit could be gained by either increasing or decreasing the hydraulic loading of the downstream cells.

Specific capacity requirements for these structures cannot be predicted with any degree of certainty. It is recommended that each of these structures be comprised of a single

10’x10’ RCB, equal to the largest of the other control structures recommended for the STA interior in order to maximize operational flexibility.

Given that size, G-382A would be capable of transferring the average rate of inflow to Cell 3 (224 cfs) to Cell 2A with a head loss of slightly over 0.1 ft. Of greater significance would be its capacity to transfer the average rate of inflow to Cell 2A (263 cfs) to Cell 3 with a headloss of just under 0.2 ft.. Upon that transfer, Cell 2B could be taken off-line, as might be desirable for the future modification of Cell 2B to support advanced treatment technologies.

Structure G-382A would be capable of transferring the average rate of inflow to Cell 2A (263 cfs) to Cell 1A with a head loss of slightly under 0.2 ft. Of greater significance would be its capacity to transfer the average rate of inflow to Cell 1A (398 cfs) to Cell 2A with a headloss of roughly 0.4 ft.. Upon that transfer, Cell 1B could be taken off-line, as might be desirable for the future modification of Cell 1B to support advanced treatment technologies.

Structures G-384A and G-384B

These structures are situated along the seepage collection canal along and parallel to the East Perimeter Levee, and serve to control the direction of seepage discharge. Structure G-384A is located at the southerly end of the seepage canal, and connects the seepage canal to the Discharge Canal. It would be closed under operations in which seepage is returned to the treatment area, and opened only when it is desired to directly discharge the seepage. The hydraulic analyses of the east perimeter seepage canal discussed earlier in this Section 8 was developed for that condition, and resulted in a computed capacity requirement of 166 cfs under a head differential of 0.5 ft. That head differential is based on a Discharge Canal (tailwater) stage of 12.1 ft. NGVD, and a headwater stage of 12.6 ft. NGVD. Given the potential for fluctuation in Discharge Canal stage resulting from the concurrent operations of S-7 and S-8, a design head differential of 0.3 ft. is recommended for this structure. It should be noted that the design capacity of 166 cfs includes 150 cfs in seepage returned from the north line of the STA at Pumping Station G-370, which could be released to the seepage canal when direct discharge is desired.

Structure G-384B consists of an existing 10’x8’ RCB beneath U.S. Highway 27, which discharges to the North New River Canal. This existing structure would be modified through addition of an operable control gate at its upstream (westerly) end. When it is desired during operation to return seepage to the treatment area, this structure would be opened (and G-384A would be closed). Seepage gathered along the east line of the STA would be discharged to the North New River Canal, and returned to the treatment area through the primary inflow pumps at G-370. Seepage gathered along the north line of the STA and pumped at G-370 would be discharged to the Inflow Canal. As a result, the peak rate of flow to be carried through G-384B is much less than is required at G-384A, as no inflow from G-370 would occur.

The tailwater stage at G-384B would be equal to the stage in the North New River Canal (which will be normally maintained at elevation 10 ft. NGVD). Given the size of the existing culvert, no appreciable head loss would be expected. The rate of seepage (and seepage capture) along the east line would increase as a result of that lower seepage collection canal stage. Given a stage of 10.0 ft. NGVD in the seepage canal, and Cells 1A and 1B at maximum design levels, the rate of discharge through this structure is estimated to be 61 cfs. The estimated head differential at that rate of flow is much less than 0.1 ft.

It should be noted that, should a determination be made not to directly discharge recovered seepage under any conditions, Structure G-384A would not be needed, and no gate would be needed at (existing) Structure G-384B.